A Case Study of Long Term Settlement of Sensitive Soft Clay Due to Embanking

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ABSTRACT: Four possible questions are newly prepared in order to solve "delayed compression/secondary consolidation" problems of natural clay. First question is on the mechanisms of the occurrence of delayed compression and/or secondary consolidation of natural clay. This difficult compression/consolidation of the clay will be shown to be quite similar to the "compaction" of loose sand, both of which proceeds with decay and/or collapse of soil skeleton structure without any significant increase in mean effective stresses. Second question is the applicability of the Asaoka's method for predicting residual settlement. In usual cases the method works, but it tends to give significantly small prediction in case of delayed compression/secondary consolidation when the method is applied at very early stages of consolidation. What sort of natural clay does exhibit difficult delayed compression/secondary consolidation? This third question will be newly solved by introducing the new "two soil indices." The fourth question is on the counter-measures against delayed compression and/or secondary consolidation. We, Japanese soil engineers, have accumulated very many bitter experiences of delayed compression/secondary consolidation for years. All the questions mentioned above will be discussed together with those severe experiences.

1 INTRODUCTON

From 1960's to late 1980's, the Japanese soil mechanics had long been occupied, for almost thirty years, by the researchers of "elastic visco-plastic theory" in order to simulate a so-called "time effect" of clay behavior. They also intended to predict "delayed compression/secondary consolidation" of natural clay in engineering practices. However, "visco-plastic constitutive models" had solved, unfortunately, extremely little, particularly for engineering purposes. After being tried in the bitter experience at the Kanda embankment site of Joban Express Way, the Japan Highway Public Corporation totally changed their design philosophy, i.e., "allowable residual settlement" totally disappeared from their official design code, which is the story of the mid 1980's, and it still continues until now (Japan Highway Public Corporation, 1998). This must be the defeat of the theoretical soil mechanics. Even now, they are saying that nobody can predict residual settlement and therefore, no soil engineers are able to be responsible for the occurrence of delayed compression/secondary consolidation. ***"What's done cannot be undone"*** must have been the design philosophy for delayed compression/secondary consolidation for more than these twenty years in Japan.

The author is introducing, in this study, the case records of road embankment problems on soft clay. However, he must state that an "accident" happened at the Kansai Int. Air Port, which was constructed on the manmade island reclaimed from the sea, cannot be an exception from this design philosophy. It is said that they spent money almost twice for the completion of this reclamation. The long lasting settlement of this airport island has already exceeded ***fourteen meters*** and not yet come to a complete stop. For this accident, however, no soil engineers and no public officials had ever been arrested before. The author is afraid, then, that the topic presented here may be beyond/behind the "forensic soil engineering."

2 WHAT IS THE POSSIBLE MECHANISM OF THE "SECONDARY CONSOLIDATION" OF NATURAL CLAY? *QUESTION 1*

2.1 *Decay/collapse of soil skeleton structure and loss of overconsolidation*

The theoretical solution for the ***QUESTION 1*** can be summarized as follows: Delayed compression/secondary consolidation occurs due to the decay/collapse of highly developed soil skeleton structure, which is, then, quite similar to "compaction/densification" of loose sand. Both proceed with no significant increase in mean effective stresses.

Naturally deposited clays/sands are mostly found in structured state and they are also more or less at overconsolidated state. The state of structure and the state of overconsolidation are both mechanical states, and they vary with ongoing plastic deformation (evolution laws). Here, the most important thing is that ***"decay/collapse of soil skeleton structure" always acts on the direction of plastic volume compression,*** while the "loss of overconsolidation" acts on the direction of plastic volume expansion. Readers may remember their intuitive interpretations like "volume compression with the card house-like collapse of soil skeleton structure" and "volume expansion due to the progressive loss of interlocking bonds between soil particles" in classical soil mechanics text books. These have been now completely solved in terms of the ***theory of elasto-plasticity*** by introducing super-sub loading yield surfaces (*see* Fig.1). Details of the super-subloading yield surface Cam Clay model (SYS Cam Clay model) can be found in Asaoka, 2003, and elsewhere.

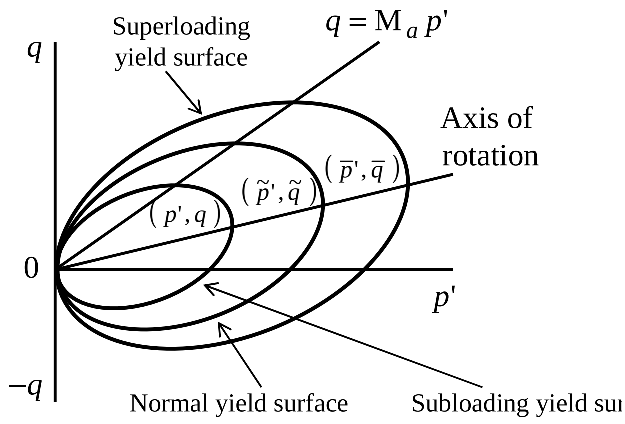

Fig. 1 Three loading surfaces in $q – p'$ space (after Asaoka et al., 2002).

When decay of soil skeleton structure and loss of overconsolidation both proceeds as plastic deformation proceeds, one may raise a simple question, i.e., "For a given rate of plastic deformation, which one proceeds faster, decay of soil skeleton structure or loss of overconsolidation?" This and only this question will clarify the difference between clay and sand, *see* Fig.2. In this figure gradation between clay and sand suggest that there exist, ***continuously***, various types of intermediate soils between them. In case of loose sand the collapse of soil skeleton structure proceeds very fast even with a small amount of plastic shear deformation, and the sudden collapse of structure in sand can be called ***compaction*** (Fig.3). (For decay/collapse of soil skeleton structure being to occur, plastic shear deformation is "ten times" more effective than plastic volumetric deformation, which is commonly true for all soils from sand to clay.) If compaction occurs under undrained condition with no volume change, elastic volume expansion should appear in order to compensate plastic volume compression. This leads to a sudden loss of mean effective stress, i.e., ***liquefaction***.

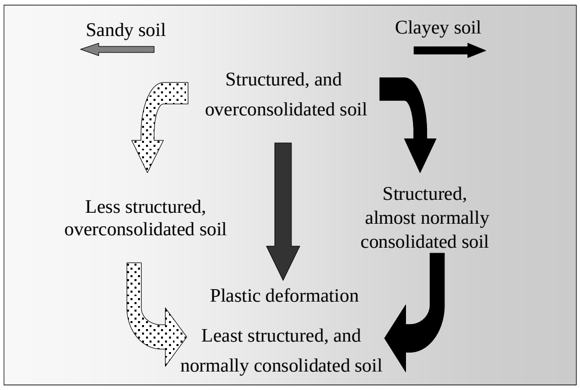

Fig. 2 Route S and route C (after Asaoka et. al., 2002).

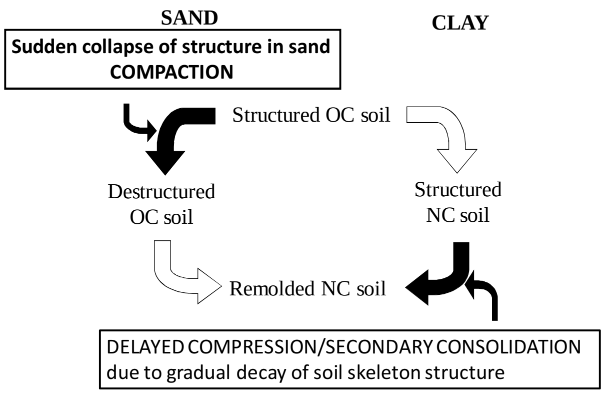

Fig. 3 Compaction of sand and secondary consolidation of clay.

In the contrary case of clayey soils, ***a near total loss of overconsolidation is followed by a very gradual decay of structure, which leads to delayed compression/secondary consolidation.*** This is also found in Fig.3. Previously, soil mechanics researchers assumed that, for the delayed compression/secondary consolidation to occur, consolidation pressure was to stride over "consolidation yield stress". This can naturally be accepted from the difference between sand and clay, i.e., clay looses its structured state after coming back to a normally consolidated state, as just stated in the beginning of this paragraph.

The SYS Cam Clay model mentioned above describes all these mechanical features of the soil consistently from sand through intermediate soil to clay by manipulating evolution laws of both structure and overconsolidation.

2.2 Joban Expressway embankment at Kanda site

Kanda embankment site on Joban Expressway is about 100km's north east of Tokyo. The embankment was constructed more than 28 years ago, but consolidation settlement has not yet come to a complete stop even now, which is shown in Fig.4. The detailed investigation and numerical analyses were only to start in these five/six years by Nagoya University Soil Mechanics Group (Noda et al., 2005), essential points of which are summarized in this section. The computer program used there is ***GEOASIA***® the name of which comes from "All Soils All states All Round ***Geo-A***naly***s***is ***I***ntegr***a***tion".

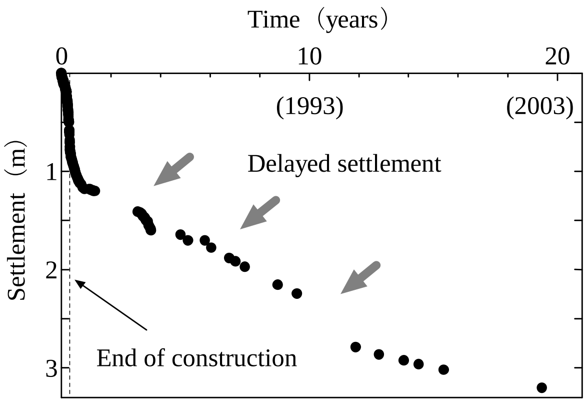

Fig. 4 Settlement behavior observed in-situ.

The embankment cross section is given in Fig.5 together with finite element array. Multilayered soft alluvial clay deposit is topped by another thick deposit of medium dense sand. Sophisticated soil tests clarified that the clay layer 2 was initially at highly structured state. Initial conditions of state variables are given in Fig.6. Material parameters of the clay and of the sand for the SYS Cam Clay model are given in Table 1.

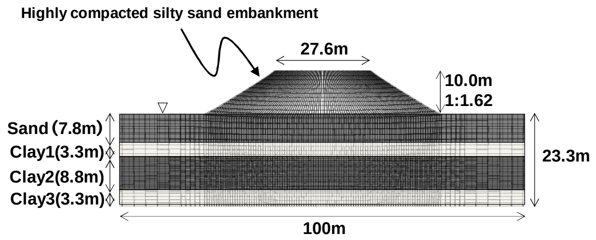

Fig. 5 Embankment and multi-layered soil foundation

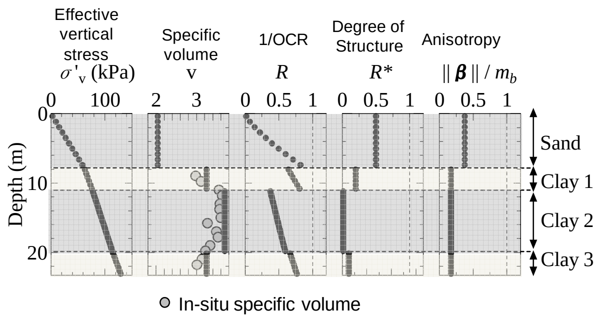

Fig. 6 Initial conditions of state variables.

Table 1. Material parameters of the clay and of the sand for the SYS Cam Clay model

		Sand	Clay 1	Clay 2	Clay 3	Embankment
Elastoplastic parameter						
Compression index	λ	0.05	0.29			0.12
Swelling index	κ	0.012	0.05			0.02
Critical state constant	M	1.00	1.90			1.3
Intercept of normal consolidation line	N	1.99	2.75			2.05
Poisson ratio	ν	0.3	0.1			0.3
Evolution parameter						
Structure degradation index	a, b, c	2.63,1.0,1.0	0.04,1.75,8.0 (/kPa)			4.0,1.0,1.0
Normal soil consolidation index	m	0.08	2.0			0.9
Totational hardening index	b_r	0.514	0.001			3.0
Critical constant of rotational hardening	m_b	0.5	0.5			0.5
Permiability coefficient	k (cm/sec)	4.0×10^{-2}	1.0×10^{-7}			1.0×10^{-4}
Soil perticle density	ρ_s(t/m^3)	2.65	2.58			2.625

(N: Specific volume on isotropic normal consolidation line of remolded clay at p'=98kN/m^2)

Numerical analysis clearly shows progressive failure of soil skeleton structure with space and with time (Fig. 7).

Typical soil element behavior during delayed compression/secondary consolidation is shown in Fig.8, which is very close to the behavior of compaction of loose sand during repeated shear stress application, *see* Fig.9.

In Fig.8, repeated increase and decrease in mean effective stress occur due to migration of pore water during progressive failure of soil skeleton structure. Migration of pore water can also be found in pore pressure behavior. Not dissipation but even rise in pore pressure was observed in situ (Fig.10), and the same is true in the results of numerical analysis (Figs. 7 and 8).

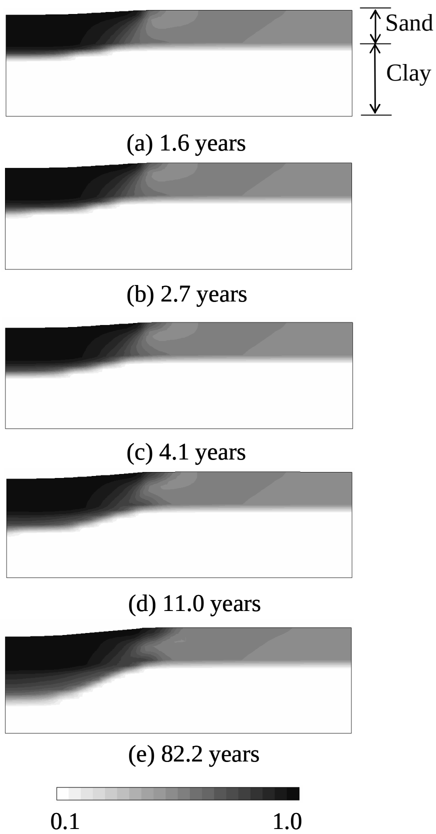

Fig. 7 Progressive failure of soil skeleton structure.

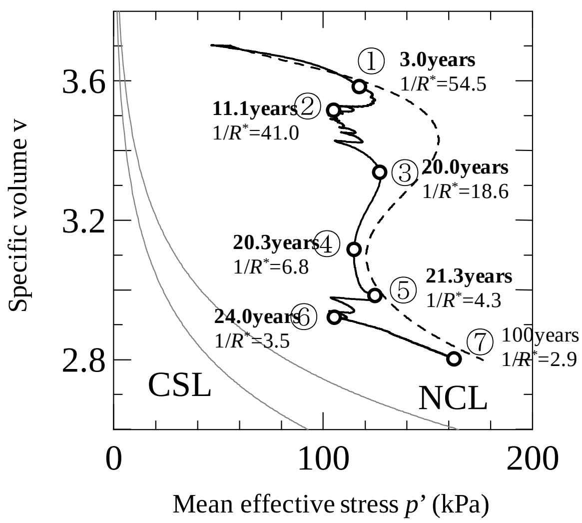

Fig. 8 Typical soil element behavior during secondary consolidation.

3 DOES ASAOKA'S METHOD WORK WELL? *QUESTION 2*

Here examined is how delayed compression/secondary consolidation looks like particularly on the Asaoka's settlement diagram, which will

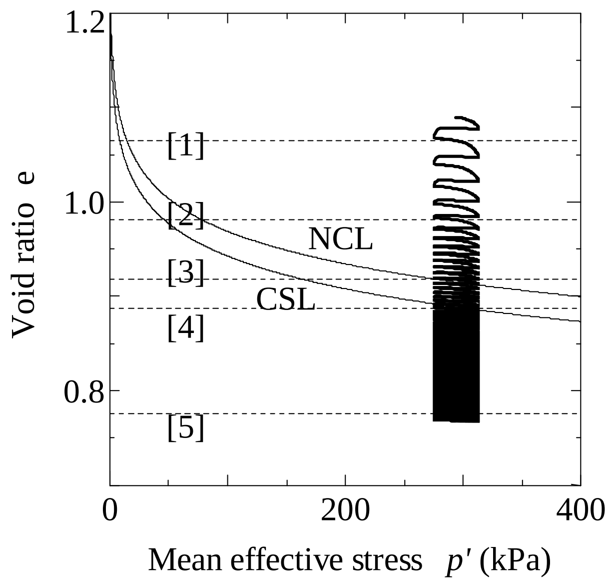

Fig. 9 Compaction of loose sand during repeated shear stress application with small amplitude (after Asaoka, 2003).

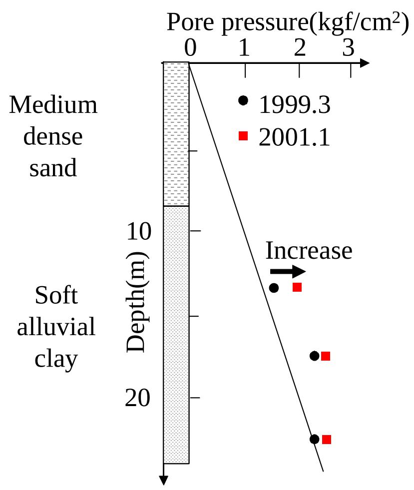

Fig. 10 Field observation of the increase in pore pressure.

clearly show advantages as well as limitations of the Asaoka's method (Asaoka, 1978).

At the Kanda embankment site, two types of construction method were comparatively examined. Embankment A was built on the original ground with no soil improvement, while Embankment B was constructed on the ground improved by "sand drain" method. As far as original soil/ground conditions are concerned, there is almost no difference between Embankment A and Embankment B. This is shown in Fig.11.

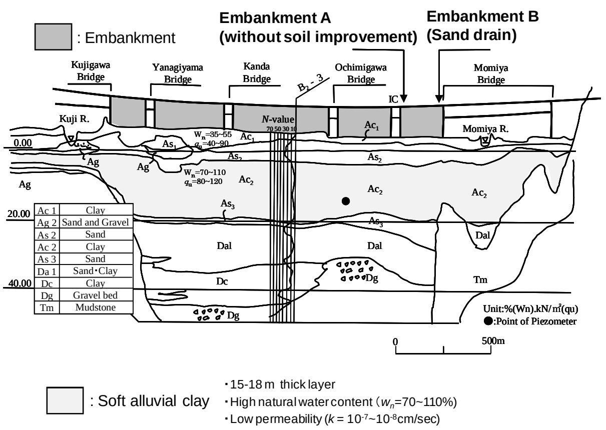

Fig. 11 Two embankments with and without sand drains.

Two embankments exhibited two distinct settlement behaviors, which were observed in situ for more than 25 years as shown in Fig.12.

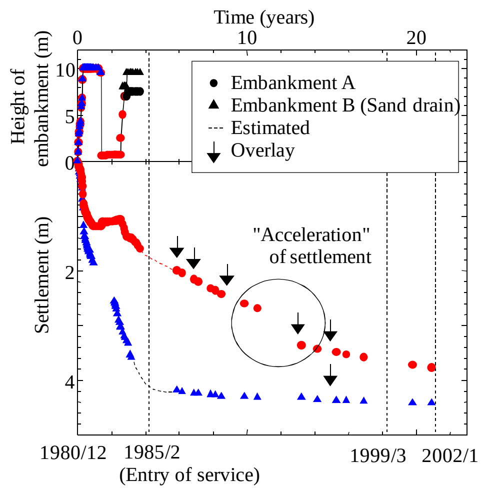

Fig. 12 Two distinct settlement behaviors.

When sand drain method is applied to the highly structured clayey ground, “cavity expansion” due to sand pile installation yields higher amount of collapse of soil skeleton structure than embankment construction works without sand drains, which causes ***higher amount of total settlement*** in sand drain areas. The collapse of soil skeleton structure due to cavity expansion proceeds under almost undrained condition, after which embanking is to start. Therefore, settlement that proceeds after embankment construction is quite similar to the ***consolidation settlement of liquefied sand layer*** after the occurrence of an earthquake. It should also be noticed that the settlement rate is accelerated with improved permeability due to sand piles. All these things are clearly observed in Fig.12.

Observational settlement prediction (Asaoka method) was first applied using settlement observations obtained during preloading procedure from 1980/10 to 1981/11. The results are shown in Fig.13.

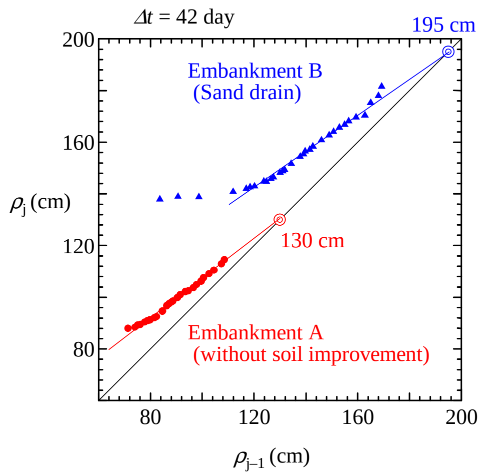

Fig. 13 Asaoka’s construction using observations at very early stages of consolidation (1980/10 to 1981/11)

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In Embankment B, it is clearly seen that Asaoka’s method cannot be applied at all for the settlement that proceeds due to collapse of soil skeleton structure. In Embankment A, decay/collapse of soil skeleton structure has not yet started. Therefore, Asaoka’s method gives a small prediction.

In order to grasp overall response of Asaoka’s method, settlement at Kanda embankment site was again computed 100 years into the future using the

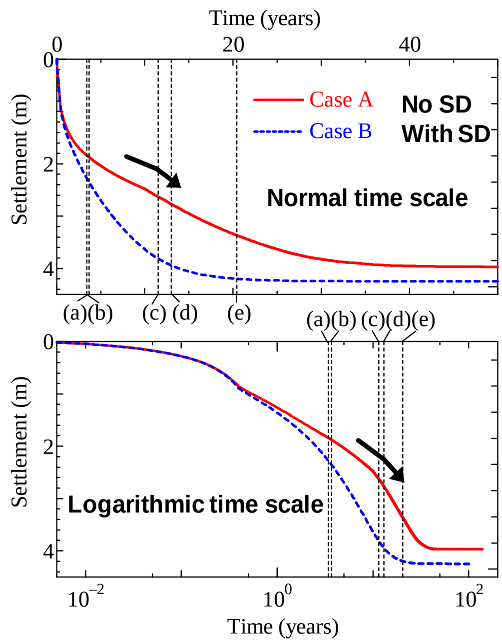

Fig. 14 Settlement behavior computed 100 years into the future.

SYS Cam Clay model (Fig. 14), from which Fig.15 follows.

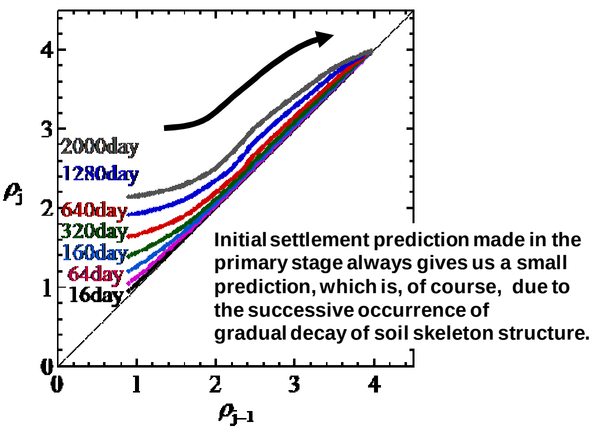

Fig. 15 Overall settlement behavior on Asaoka's diagram before and after decay of soil skeleton structure.

As can be seen in Fig. 15, initial settlement prediction made in primary stages of consolidation always gives us a small prediction.

4 IS IT POSSIBLE TO DISTINGUISH DIFFICULT CLAY FROM THE OTHERS? *QUESTION 3*

4.1 *Many bitter experiences in Japan*

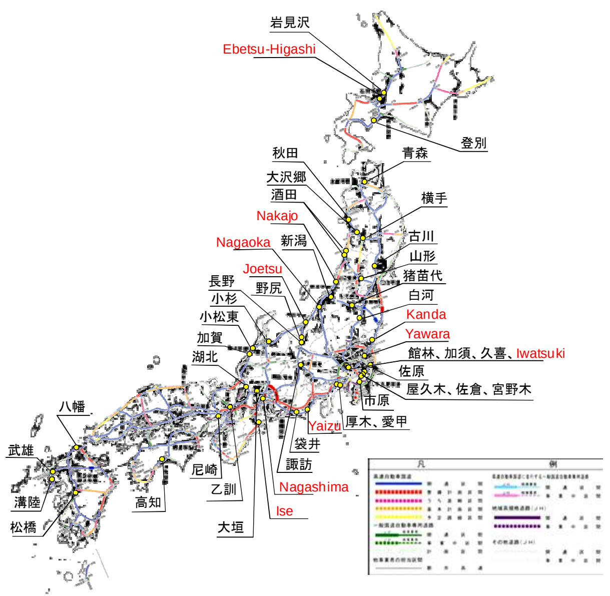

Fig. 16 More than 50 road embankments on soft alluvial clay deposits.

Japan Highway Public Corporation has ever constructed more than 50 road embankments on soft alluvial clay deposits (Fig.16). Among which, more than 20% of the embankments have exhibited miserable residual settlement of more than 1m or more after entry of services.

Some examples are shown in Figs.17 – 19, and Table 2. As seen in this table, embankment height and thickness of clay layer alone are not sufficient parameters to distinguish difficult sites from the other normal sites.

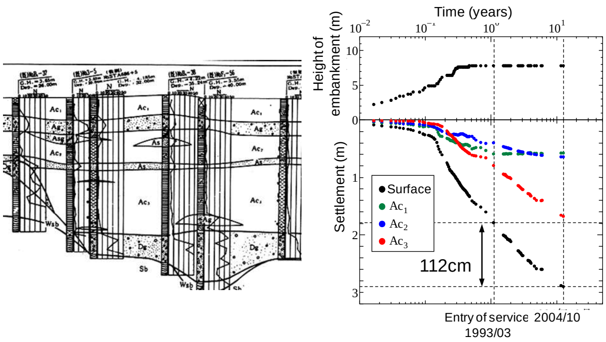

Fig. 17 Experience in Ise (Site H).

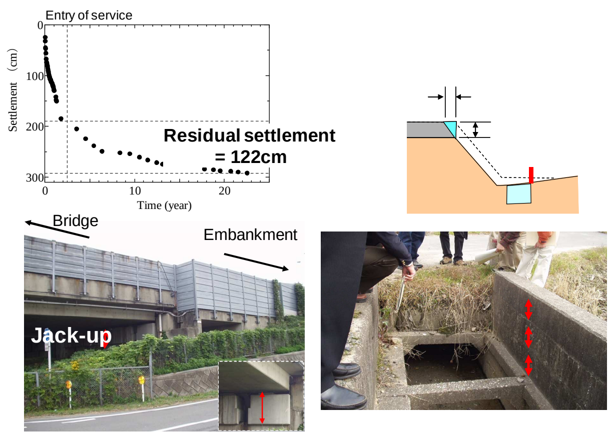

Fig. 18 Experience in Nagashima (Site J).

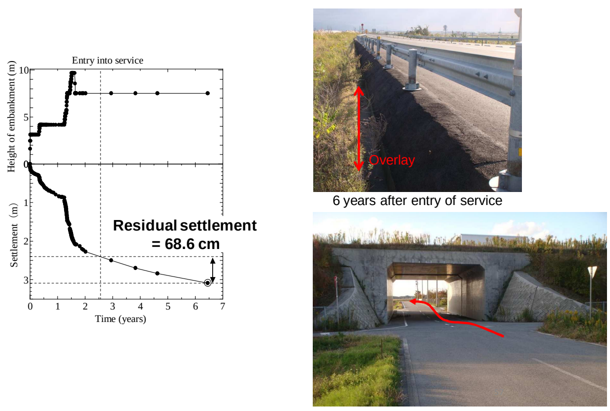

Fig. 19 Experience in Nakajo.

Table 2. Various performance of road embankment works.

Site	Height of embank-ment (m)	Thickness of soft clay layer (m)	Residual settlement (cm)	Measuring point after the entry of service (year)
<Small residual settlement>				
A	5.7	30	**30**	20
B	7.7	28	**67**	20
C	7	23	**38**	20
D	6.8	14	**29**	20
E	6	13	**55**	20
F	9	11	**26**	10
G	10	10	**3**	10
<Large residual settlement>				
H	8	25	**112**	12
I	9	16	**188**	16
J	7.5	31	**122**	20
K	5	22	**200**	25

4.2 *The two important indices of natural clay*

Nagoya University Soil Mechanics Group newly proposed two soil indices in order to distinguish difficult clay from the others (Inagaki et al., 2010). Difficult clay is the clay that exhibits delayed compression/secondary consolidation when applied load is striding over "consolidation yield stress". First soil index is the sensitivity ratio S_t and the second one is the compression index ratio *Cc*/*Ccr*.

Sensitivity ratio given in Fig.20 clearly indicates how bulky the clay is, i.e., the degree of soil skeleton structure. Bigger the sensitivity ratio, the lager the delayed compression becomes.

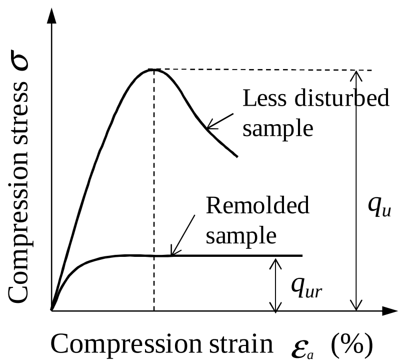

Fig. 20 Sensitivity ratio S_t $(=q_u/q_{ur})$.

On the other hand, "compression index ratio *Cc*/*Ccr*" the definition of which is given in Fig.21 will show how rapidly the collapse of soil skeleton structure proceeds. Bigger the compression index ratio, more easily secondary consolidation proceeds even with a small amount of applied load.

When fully remolded samples are not available, Skempton's estimate,

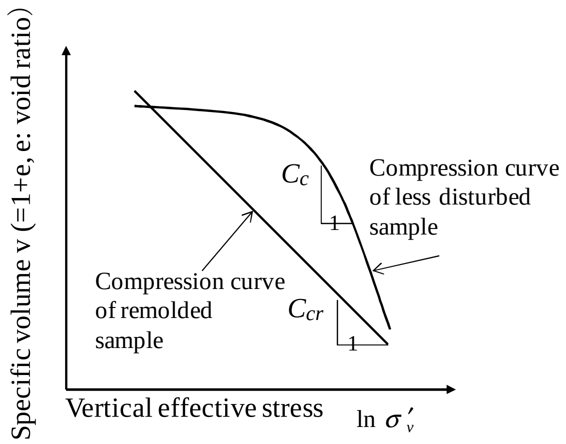

Fig. 21 Compression index ratio=*Cc*/*Ccr*.

$$Ccr = 0.007(w_L - 10) \qquad (1)$$

may be helpful, in which w_L is a liquid limit (%).

Many of the experiences that have been accumulated so far by Japan Highway Public Corporation are plotted in a single S_t - *Cc*/*Ccr* diagram (Fig. 22).

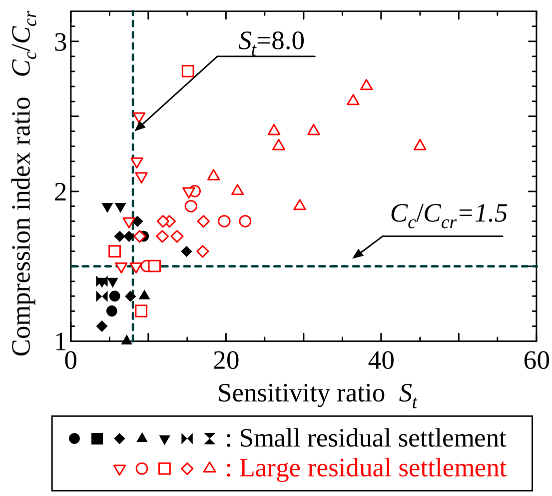

Fig. 22 S_t - *Cc*/*Ccr* diagram.

Readers will recognize that difficult soils can be clearly distinguished from the others when border lines for the distinction are introduced in the S_t - *Cc*/*Ccr* diagram as follows:

$$S_t > 8.0, \quad \text{and} \quad Cc/Ccr > 1.5 \qquad (2)$$

The typical two distinct soil behaviors are shown in Fig.23.

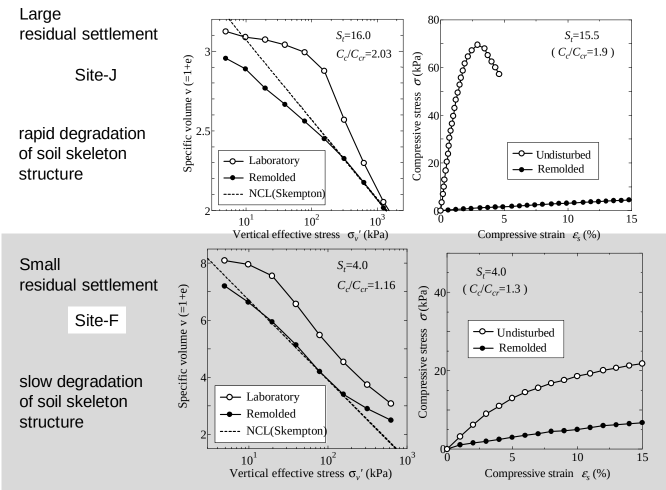

Fig. 23 Typical two distinct soil behaviors.

The Japanese *"theoretical"* elasto-plastic soil mechanic has long been led by the people who learned things from the classical Cam Clay model. Most of them spent their time in Cambridge University in 1970's and early 1980's. The Cam Clay model is, however, no more than the model of the fully remolded artificial clay. This must be the possible reason why Japanese Geotechnical Society (JGS) has not yet established, even now, their official testing standard for measuring "sensitivity ratio S_t" of natural clay. Fig.24 is prepared for this reason.

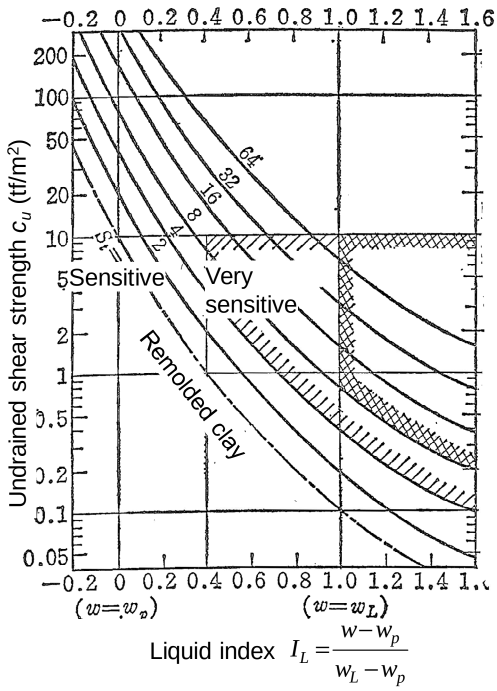

Fig. 24 Mikasa's diagram for estimating sensitivity ratio S_t

Again, when fully remolded samples are not available, it is impossible to get sensitivity ratio S_t. In such cases, the sensitivity ratio can roughly be estimated from undrained shear strength c_u and liquid index I_L

They say that Fig. 24 was first established by Professor M. Mikasa of Osaka City University in late 1960's.

5 WHAT ARE THE POSSIBLE COUNTERMEASURES? TO WHAT EXTENT ARE THEY EFFECTIVE? *QUESTION 4*

In Kanda embankment site on Joban Expressway, "Sand Drain Method" was examined by comparing the case without sand drains. Fig.12 clearly shows advantages and limitations of the "Sand Drain Method," the discussion of which has already been given briefly/essentially in section 3.

Another case record comes from Nakajo Site on Nihonkai-Tohoku Expressway (*see*, Fig.19). In this site, contrary to Embankment B in Kanda site, embanking was carried out without the use of any artificial drainage method for the original clay stratum like sand drains. This is because the soft clay layer in the site was topped by a thick sand deposit on it. Residual settlement has already reached 70cm in the first 4 years after the entry into service, *see* Fig.25.

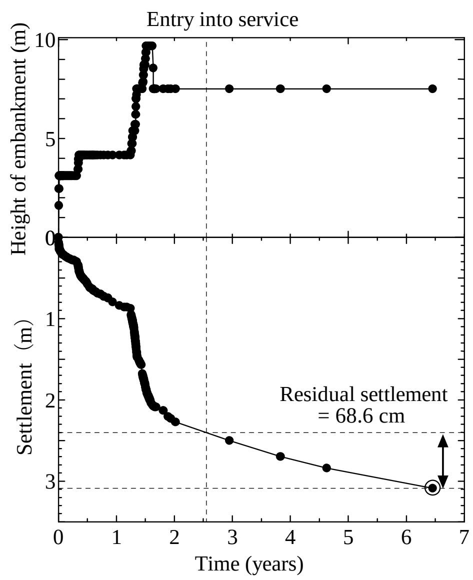

Fig. 25 Settlement observation at Nqakajo embankment site on Nihonkai Tohoku Expressway

The clays at Nakajo site were found to have had a high potential for the occurrence of large residual settlement/secondary consolidation, which can be clearly seen when the "S_t – *Cc/Ccr* diagram" is applied (Fig.26) to this site, although the diagram was not available at that time.

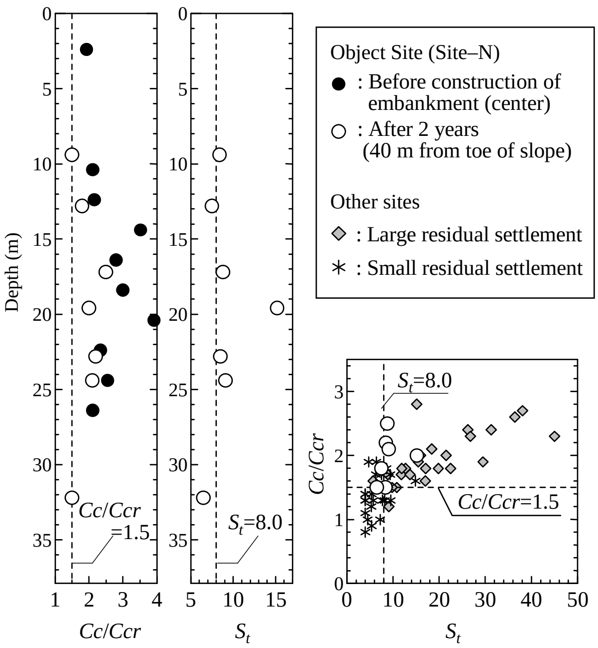

Fig. 26 Severe soil conditions at Nakajo site found on the S_t - *Cc*/*Ccr* diagram

Numerical analysis by ***GEOASIA***® was again asked for solving the problem in this site (Fig.27).

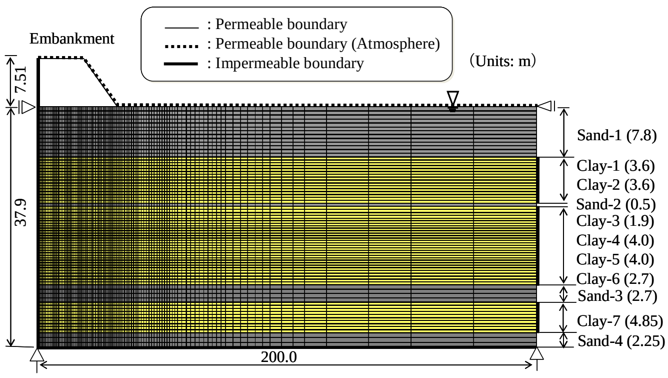

Fig. 27 Complex soil conditions at the Nakajo site

In the first stage of analysis, the settlement behavior observed for the first five years was simulated up to the present time, through which necessary but unknown/uncertain soil parameters are definitely identified. Future behavior of the clay foundation is then predicted using the same parameters/soil conditions, which is nothing but the ***observational procedure by Terzaghi***. For the detailed analysis method and its performance, *refer* to Tashiro, et al. (2011).

Predictive computation was made 100 years into the future, which is shown in Fig.28.

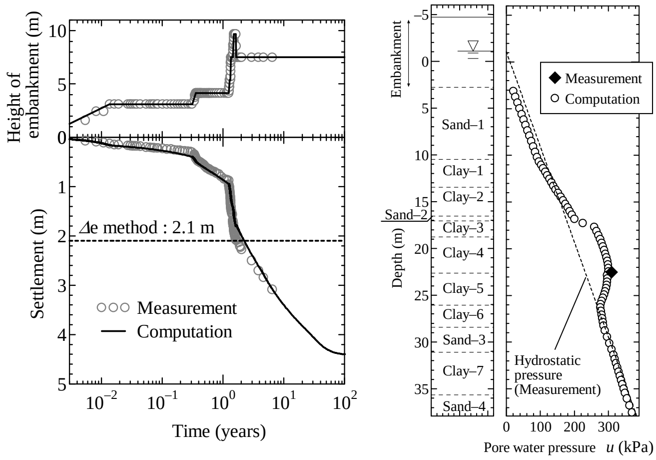

Fig. 28 Settlement behaviour at Nakajo site with the predictive settlement 100 years into the future

Counter measures that are possible even after the finish of embanking and/or after the road service are quite limited. The two alternatives were examined in this site. First one is the ***"overlaying"*** and the other, ***"lightweight banking method"***. In *"overlaying,"* 30-cm thick overlays are continuously repeated for every 30cm of settlement. In *"lightweight banking method"* they were intending to replace the existing embankment with light weight embankment material such as EPS and others.

The two alternatives as a possible countermeasure for this site were numerically compared, essential points of which are shown in Figs.29 and 30. Although the total settlement would reach 3m's, they decided ***"overlaying"***. They are now expecting, in this site, 30cm thick overlays to be repeatedly carried out 10 times over the next 60 years!

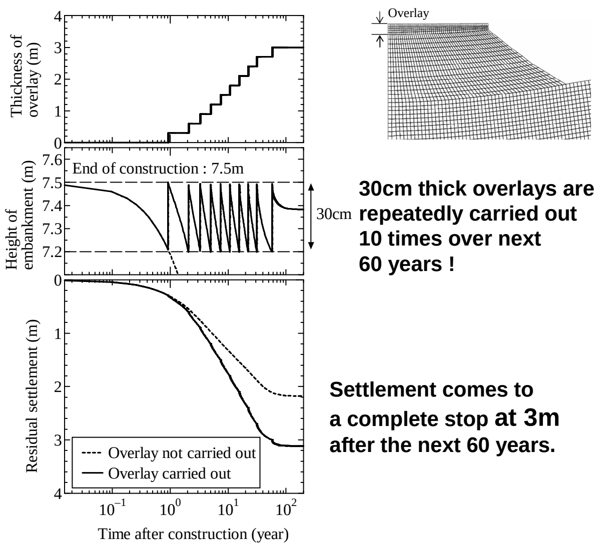

Fig. 29 Overlaying method

The ***"lightweight banking method"*** was found less effective (Fig.30). Once the collapse of soil skeleton structure started, nothing can be done! Remember "What's done cannot be undone*!?*"

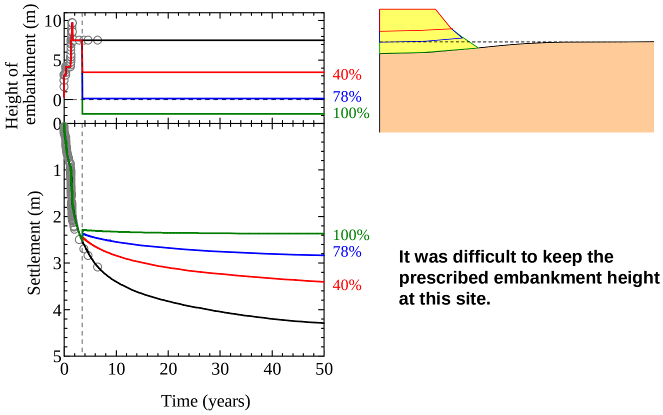

Fig. 30 "Lightweight banking method" that replaces original banking materials with reduced weight materials after the finish of embanking.

6 CONCLUSION

Delayed compression and/or secondary consolidation of natural clay was quite similar to the "compaction" of loose sand, both of which proceed with progressive failure of soil skeleton structure with time and with space, which is possible to occur without any significant increase in mean effective stresses/compressive effective stresses. This will be the fact that the theory of visco-plasticity is difficult to clarify.

The conventional elasto-plasticity theory, like Cam-Clay model, had also long been difficult to solve the problem before they newly got the conception of super-subloading yield surfaces. These loading surfaces describe the decay/collapse of soil skeleton structure as well as the loss of overconsolidation consistently, both of which proceed as plastic deformation proceeds.

The Asaoka's method for predicting residual settlement tends to give significantly small prediction in case of delayed compression/secondary consolidation, when the method is applied at very early stages of consolidation.

Natural clay that exhibits difficult delayed compression/secondary consolidation is preferable to be distinguished from the others beforehand, for which purpose the use of "sensitivity ratio" and "compression index ratio" is newly recommended.

"What's done cannot be undone" has long been a single design philosophy for this problem for years until now in Japan. Even the "accident" at the Kansai Int. Air Port cannot be an exception, because this accident has long been outside the so-called "forensic" matters.

Soil mechanics and/or geotechnical engineering has been regarded as a typical "empirical technology." Even now it is occasionally said that geotechnical engineering is belonging not to the "sci- "science" but to the "art." However, not art but science must be expected to solve soil engineering problems, particularly after the bitter experiences in "Tohoku" and "Fukushima."

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